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Erector Connector Meadow Burke Company In-Plane Performance

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ERECTOR CONNECTOR
MEADOW BURKE COMPANY
IN-PLANE PERFORMANCE

FINAL REPORT

By

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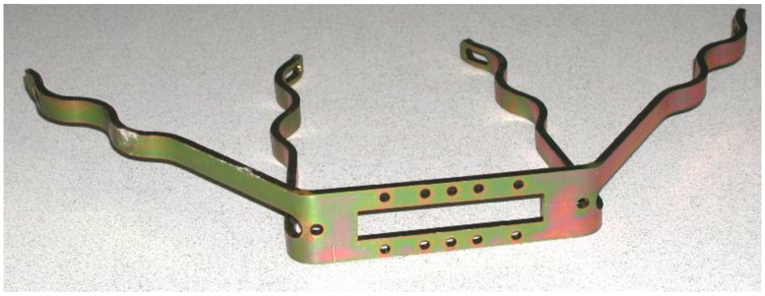
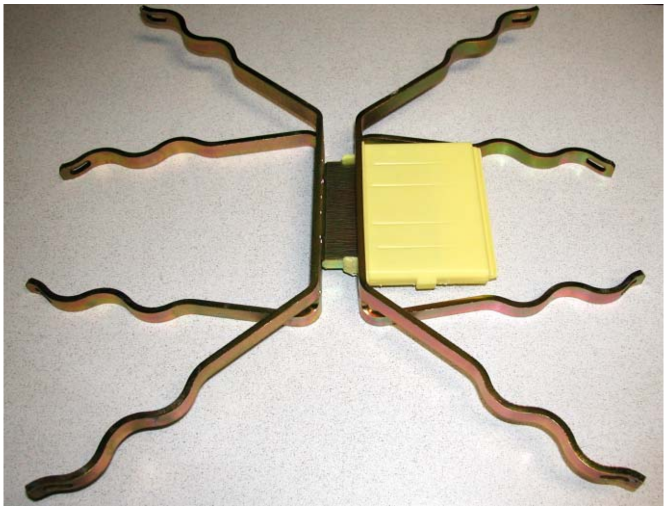
ABSTRACT

This report summarizes the in-plane performance of the Erector Connector developed by Meadow Burke Company. The connector is intended for use as flange-to-flange connectors between precast double tee panels with 4-in. flanges. The connector was tested under monotonically increasing shear, monotonically increasing tension, cyclic shear, and shear with proportional tension. The resulting capacities and associated damage are summarized in the report. This work was funded by Meadow Burke Co. and was conducted at the ATLSS Center at Lehigh University.





BACKGROUND

As a means of assessing the displacement capacity and structural stiffness of connections in precast diaphragms, an experimental study was conducted. A subassembly consisting of the connector and a portion of the surrounding diaphragm was developed. The subassemblies include a Type A connector and Type B connector embedded in standard 4-in. precast concrete panels. All specimens were fabricated at full-scale. This report summarizes the experimental results of the Erector connector tested under displacement control in monotonic tension, monotonic shear, cyclic shear and combined shear with tension.

Subassembly Details

The subassembly was developed assuming that the connectors are spaced at 4 feet and embedded in a double tee panel with a 2ft distance from the DT web to the free flange face. The test specimens are fabricated from two panels 2ft wide and 4ft long (Figure 1). The panels are connected to form a 4ft square subassembly. Welded wire reinforcement (WWR) is included in each panel to meet ACI temperature and shrinkage reinforcement requirements. In addition to the WWR conventional reinforcement is used to maintain integrity during testing. The bars are placed at the periphery of the panel to minimize influence on the connector response. The supplemental reinforcement is illustrated in Figure 3.

The Erector Connector developed by the Meadow Burke Company is evaluated. The connector (Figure 2) is designed for placement in 4-in. double tee flanges. It consists of two connector types A and B. Type A consists of a bent plate connector with four anchor legs, two oriented at 45-degrees and 2 and 90-degrees from the face plate. A slug is included with the connector which allows it to be pulled out once the DT members are in place. Type B has a similar configuration to type A, but does not include the removable slug. Both connectors are fabricated from ASTM A36 steel and welded to the adjacent panel the embedded Type A ASTM A36 steel slug and a E7018 weld electrode. The welds were conducted at room temperature using a SMAW process according to AWS specifications. Variations on welding were used in some cases a weld was placed only at the connection B face and in other cases at both the B and A faces.

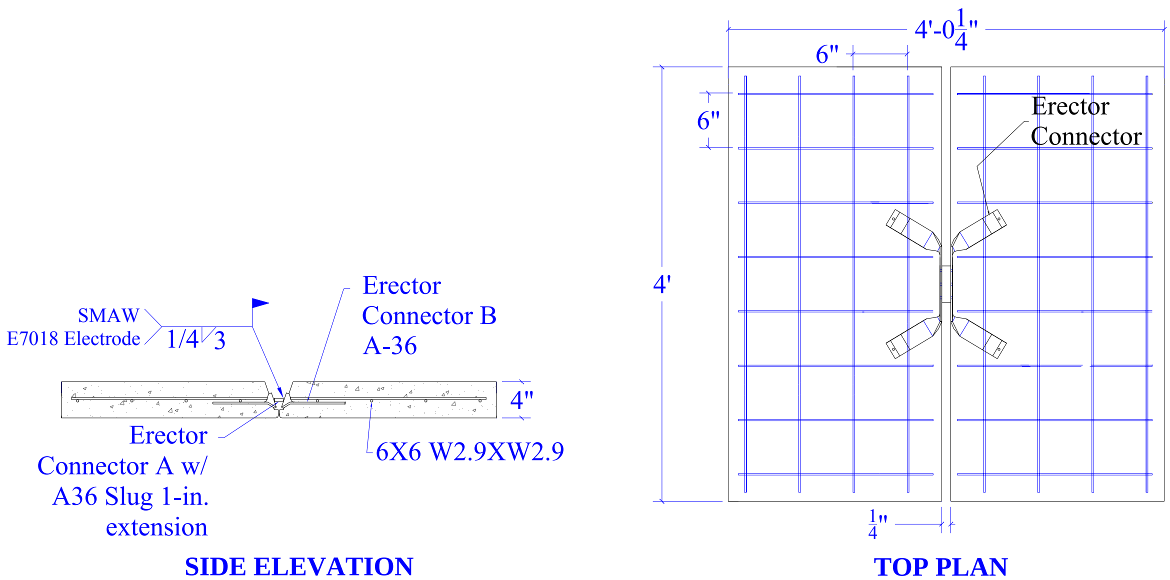


Figure 1: Specimen details

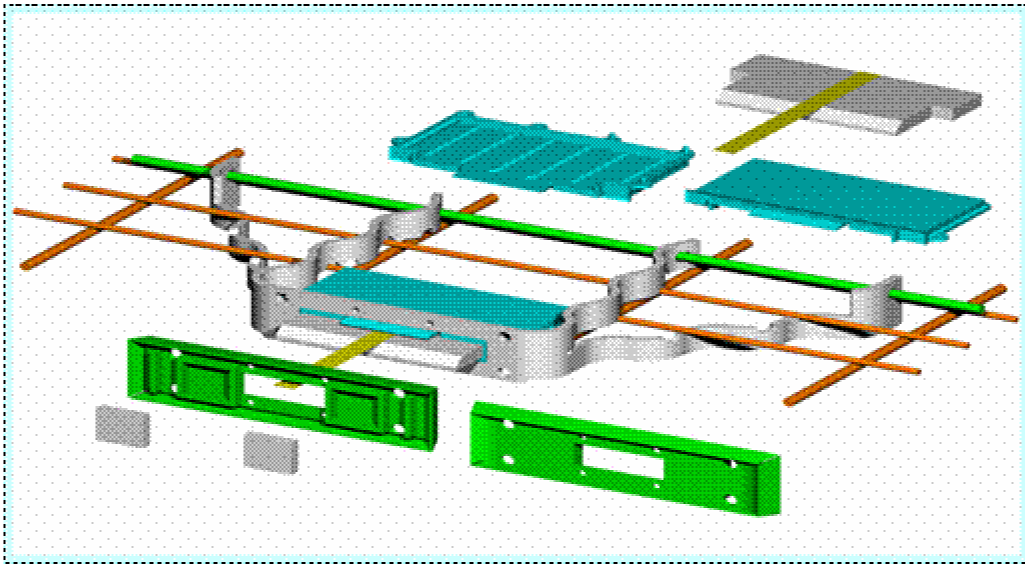


Figure 2: Type A Erector Connector

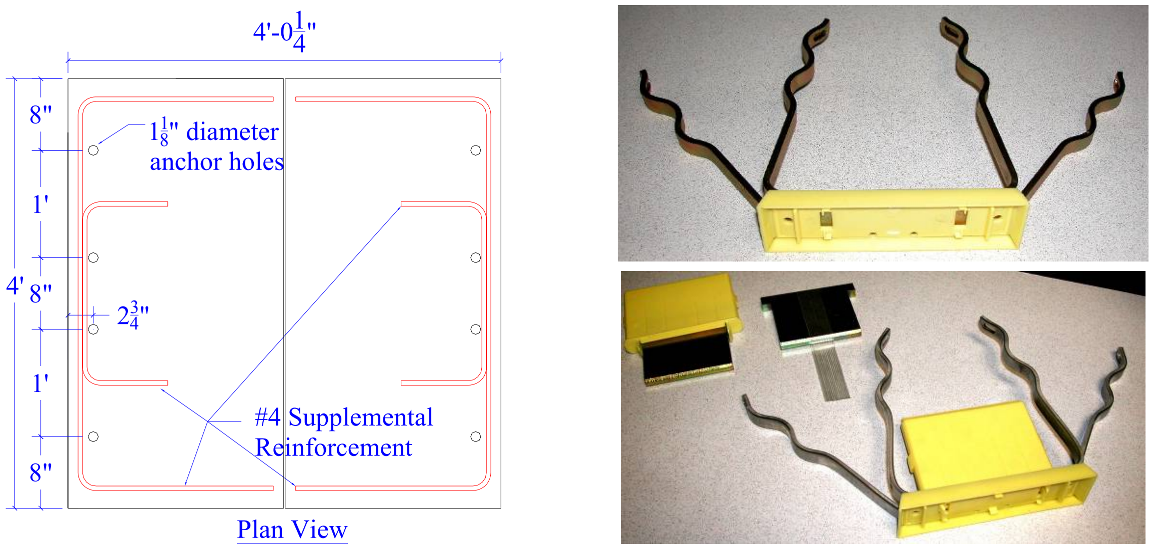


Figure 3: Supplemental reinforcement layout and construction details

Deformation Protocols

The connector was evaluated under in-plane shear, tension, and combined shear with tension. All tests were conducted under quasi-static displacement control at a rate less than 0.05in/sec. The tests were continued until failure. Failure is defined as the point where the specimen capacity drops below 25% of the measured ultimate. Five displacement protocols have been developed to represent the spectrum of demands a local diaphragm connector could experience under lateral loading [Naito 2005]. Three of these deformation protocols were used in the current study:

1. Monotonic Shear
2. Cyclic Shear
3. Monotonic Tension
4. Monotonic Shear with Proportional Tension

Monotonic Shear

The monotonic shear tests were conducted to evaluate the connector response under pure shear deformation. The original panel separation of was maintained through the test. The test represents the joint condition where the panels are shearing without any flexural opening or closing. The test thus provides an estimate of average connector yield, peak strength, and the deformation capacity. Monotonic shear protocol consists of three cycles to 0.01-in. to estimate initial stiffness and verify equipment operation. Afterwards, the specimens were loaded monotonically to failure (Figure 4).

Cyclic Shear

Cyclic shear tests provide insight on the degradation of shear properties (i.e., stiffness and ultimate strength) under loading reversals. The loading protocol is based on the PRESSS program [Priestley 1992]. Three preliminary cycles to 0.01-in. are conducted to evaluate control and acquisition accuracy. The remaining protocol consisted of groups of three symmetric shear cycles at increasing deformation levels. Each level is based on a percentage of a reference deformation computed from the preceding monotonic test. The reference deformation represents the effective yield deformation of the connector. It is computed by taking the intercept of a horizontal line at the max load and a secant stiffness line at 75% of the max load (Figure 4 inset). Three elastic levels of 0.25⊿, 0.50⊿ and 0.75⊿ followed by inelastic cycles to 1.0⊿, 1.5⊿, 2.0⊿, 3.0⊿, 4.0⊿, 6.0⊿, 8.0⊿, *etc…* were conducted. The loading protocol is illustrated in Figure 4.

Monotonic Tension

In current diaphragm design, the flexural diaphragm tensile forces are assumed to be resisted by the chord reinforcement. The contribution of shear connectors to flexural resistance is commonly neglected. Previous research has shown that in many cases web connectors provide high tension stiffness. To quantify the relative tensile contribution of the web connectors and chord connectors, a monotonic tension tests were conducted. The loading protocol consisted of three tension/compression deformations to 0.01-in. followed by a monotonically increasing tension deformation to failure (Figure 5). The test was paused at each 0.1-in. for observations.

Monotonic Shear with Proportional Tension:

The monotonic shear with tension test consists of three cycles of 0.01-in. in shear and a proportional tension/compression deformation. The shear and tension deformations will be increased proportionally using the chosen constant shear-to-tension deformation ratio of 2.0. The test will be paused at each 0.1–in of shear deformation for observations. The test is performed with the initial joint opening maintained through the test.

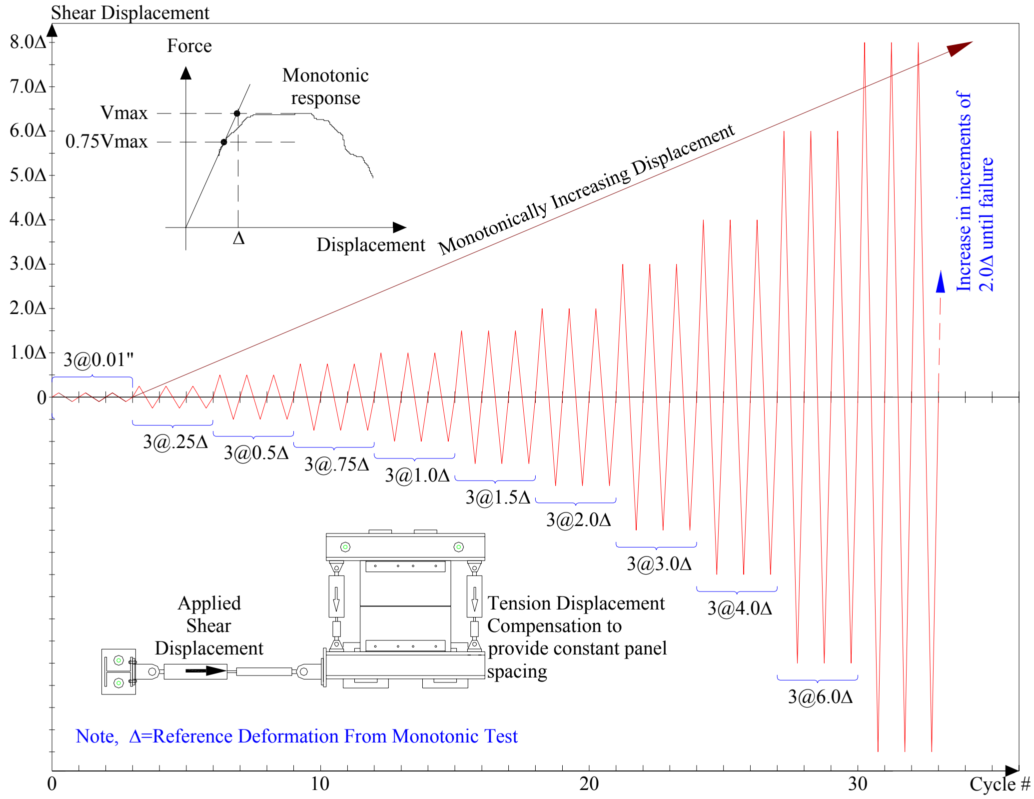


Figure 4: Shear loading protocol

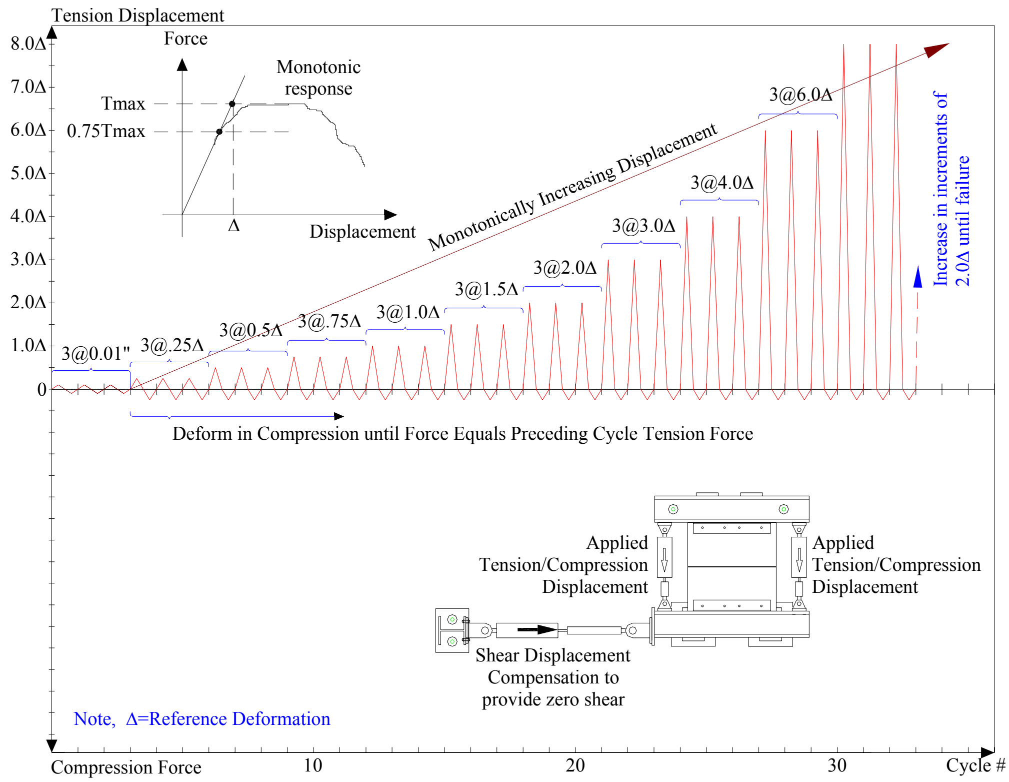


Figure 5: Tension/Compression protocol

A total of six tests were performed on the connector. In each case one type A connector was welded to one Type B connector. In some cases one side was welded and in other cases both sides were welded. For the single weld tests the embedded slug was pulled out of connector A and welded to connector B. For the double weld tests an additional weld was applied to connector A to secure the embedded slug to the faceplate of the connection. Each weld consisted of a 3in. long 1/4in. filet weld. The test designations and the type of test are presented in Table 1.

Table 1: Test matrix		
Designation	Weld Configuration	Description
A1	Both sides welded	Monotonic tension with no shear force
A2	Both sides welded	Monotonic shear with no tension deformation
A3	One side welded	Monotonic shear with no tension deformation
A4	One side welded	Cyclic shear with no tension deformation
A5	Both sides welded	Cyclic shear with no tension deformation
A6	Both sides welded	Shear with proportional tension deformation ($\Delta V/\Delta T = 2.0$)

Material Properties

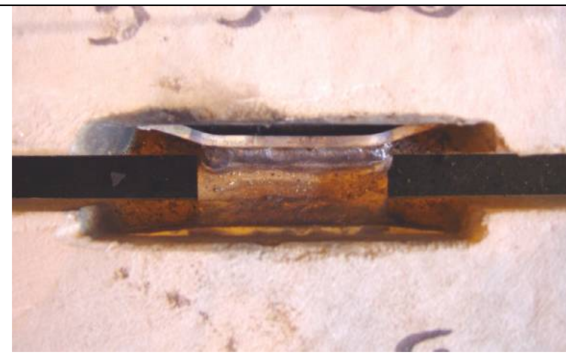
The base 4-in. precast panels were fabricated using ready mix concrete with design 28-day strength of 5000 psi. The WWR used in the base panel met the requirements of ASTM A185 grade 65 steel. The connectors were furnished by Meadow Burke. Material data supplied with the connectors indicated that the Erector connector was fabricated from A-36 steel, plate properties were not available. The slugs used to connect the panels are included in the Type A connector and were fabricated A-36 steel. The measured concrete strengths and mill certified steel properties are presented in Table 2.

Table 2: Material Properties				
Size	Reinforcement Usage	ASTM Grade	Yield Stress (ksi)	Ultimate Strength (ksi)
Embedded	Connector Slug	A36	36*	50*
Bent plate	Connector	A36	36*	50*
#4	Reinforcing Bars	A615	90*	106*
W2.9XW2.9 6X6	Precast Panel Mesh	A185 Gr.65	65.00*	107.0
* Data unavailable, value assumed				

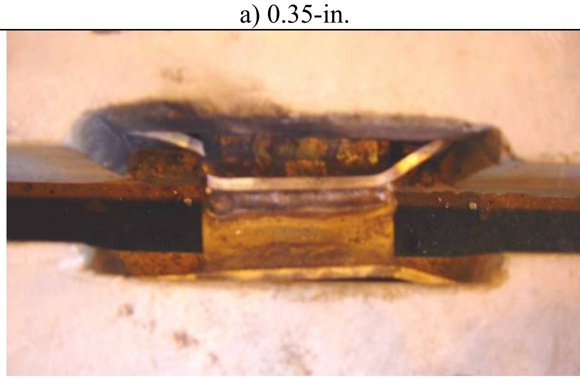
TEST A1: ERECTOR CONNECTOR UNDER MONOTONIC TENSION WITH NO SHEAR FORCE

The performance of the Erector connector subjected to monotonic tension is presented in this section. ***The connector is welded on both side A and B***. The panel was subjected to tension displacement with the shear force unrestrained, Fv=0. Connector damage initiated with faceplate bending of connector A. This placed large tensile demand on the top faceplate of the Type A connector. This resulted in high tension demands on the thin section above the faceplate opening adjacent to the weld. This tension demand resulted in fracture of the face plate at each end of the weld at 0.62-in. and 1.22-in. of opening displacement. A third increase in strength occurred as the embedded plate tabs contacted the opening of connector A. The slug pulled out at approximately 2-in. of deformation. A complete loss in resistance occurred after pullout of the plate. The observed key events and the corresponding displacement level are presented in Table 3. The photos of the damage are presented in Figure 6 and Figure 7. The global force deformation response and backbone curve are presented in Table 4 and Figure 8.

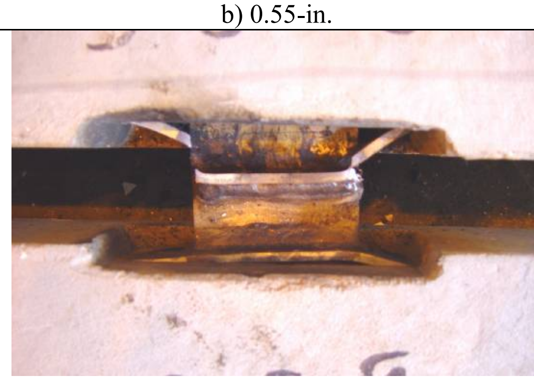
a) 0.35-in.



b) 0.55-in.



c) 1.10in.



d) 1.35-in.

Figure 6: Damage state at various tensile openings

a) 

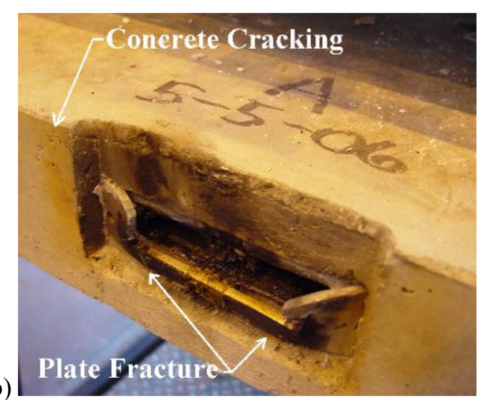
b) 

Figure 7: Damage at end of test a) Connector B, b) Connector A

Table 3: Key Test Observations (Monotonic Tension)		
Event #	Tensile Δ Step [in.]	Event Description
1	0.20	Face plate bending from connector A
2	0.85	Fracture of face plate of connector A near left side of weld
3	1.10-1.35	Fracture of face plate of connector A near right side of weld
4	2.1	Pullout of tab from box of connector A
5	3.10	Test Stopped

Table 4: Experimental Results Backbone Curve (Monotonic Tension)		
Event	Tensile Displacement [in.]	Tensile Force [kips]
Elastic Limit	0.104	1.80
-	0.145	2.28
-	0.177	2.44
Peak Load	0.518	3.59
-	0.624	3.50
-	0.668	1.40
End of test	2.35	0

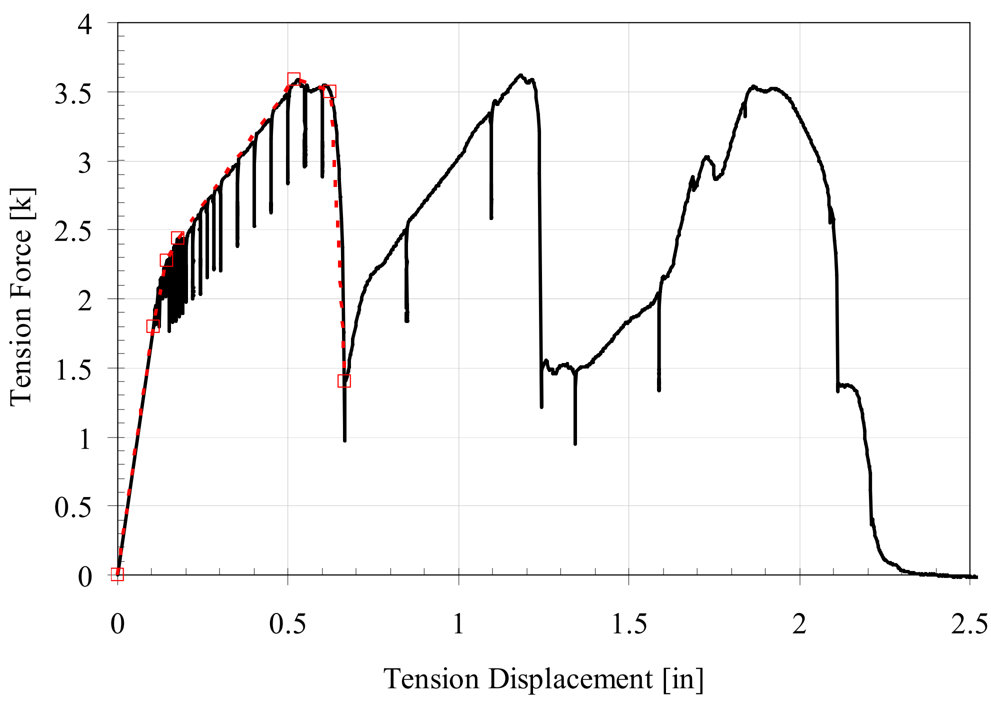
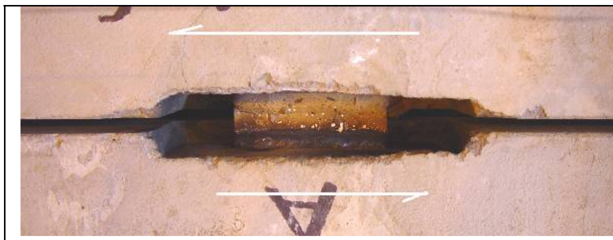


Figure 8: Force and axial displacement (Monotonic Tension)

TEST A2: ERECTOR CONNECTOR 1 UNDER MONOTONIC SHEAR DEFORMATION WITH $\Delta T = 0$

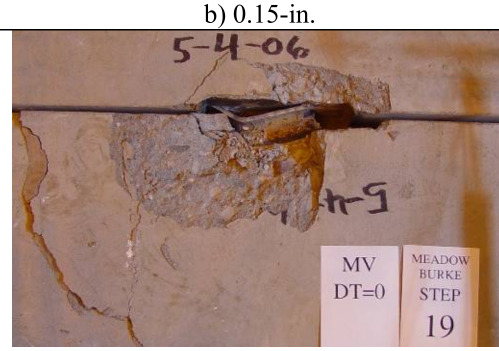
The performance of the Erector Connector subjected to monotonic shear is presented in this section. ***The connector is welded on BOTH side A and B***. The panel was subjected to shear displacement with the tensile displacement restrained, $\Delta T=0$. Connector damage initiated with rotation of the front face plate of connector B which continued with additional shear deformation. Crushing and spalling occurred above the compression leg of connector A. This was followed by similar spalling damage over the compression leg of connector B. The amount of spalling increased on connection A. Fracture of connector A occurred on the tension leg. However even after the tension leg was lost the compression legs were capable of resisting shear through bearing of the embedded slug. The observed key events and the corresponding displacement level are presented in Table 5. The photos of the damage are presented in Figure 9. The global force deformation response and backbone curve are presented in Table 6 and Figure 10.



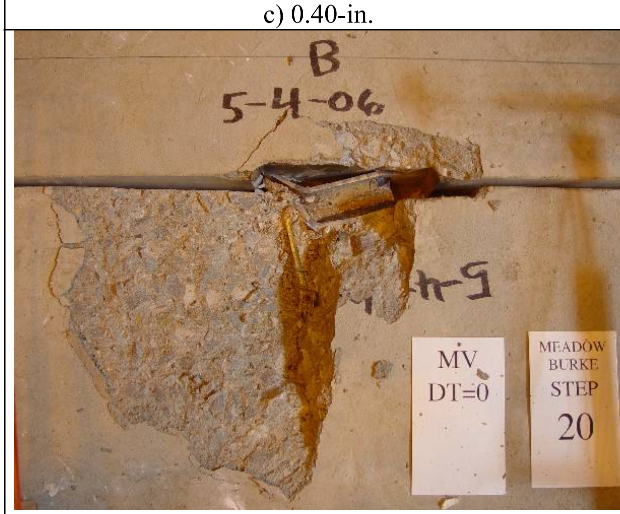
a) 0.02-in.

b) 0.15-in.

c) 0.40-in.



d) 1.25-in.



e) 1.5-in.


f) 3.0-in.

Figure 9: Damage state at increasing shear deformations

Table 5: Key Test Observations Erector Connector 2 Welds (Monotonic Shear)		
Event #	Shear Δ [in.]	Event Description
1	0.25	Cracking audible
2	0.30	Panel A compression leg cracking and associated spalling
3	1.00	Spalling over tension leg on panel B
4	1.25	Tension failure observed on connector A and compression pushout damage
5	1.50	Additional pushout of compression side panel A
6	2.50	Cracking over panel A tension side
7	3.00	Test Stopped

Table 6: Experimental Results Backbone Curve 2 Welds (Monotonic Shear)				
Shear force – Shear deformation			Axial force – Shear deformation	
Step	Shear Displacement	Shear Force	Shear Displacement	Axial Force
	11.88	0.09	11.88	-3.01
First Peak	20.24	0.26	20.24	-6.88
	14.64	0.41	14.64	-5.09
	23.07	1.06	23.07	-10.81
Secondary Peak	23.42	1.19	23.42	-11.28
	5.47	1.47	5.47	-2.13
Failure of Connector	10.87	3.00	10.87	-2.33

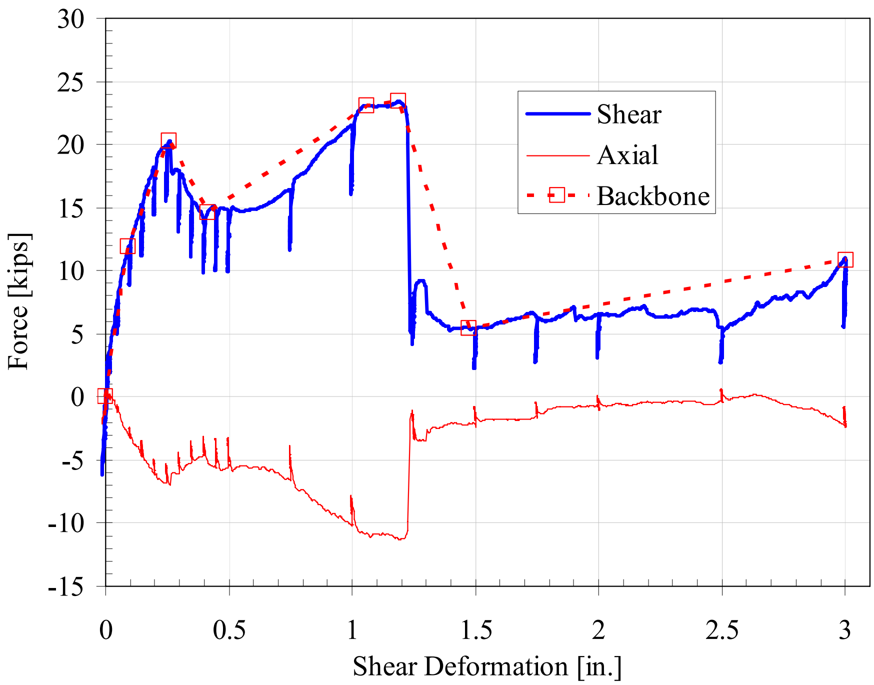
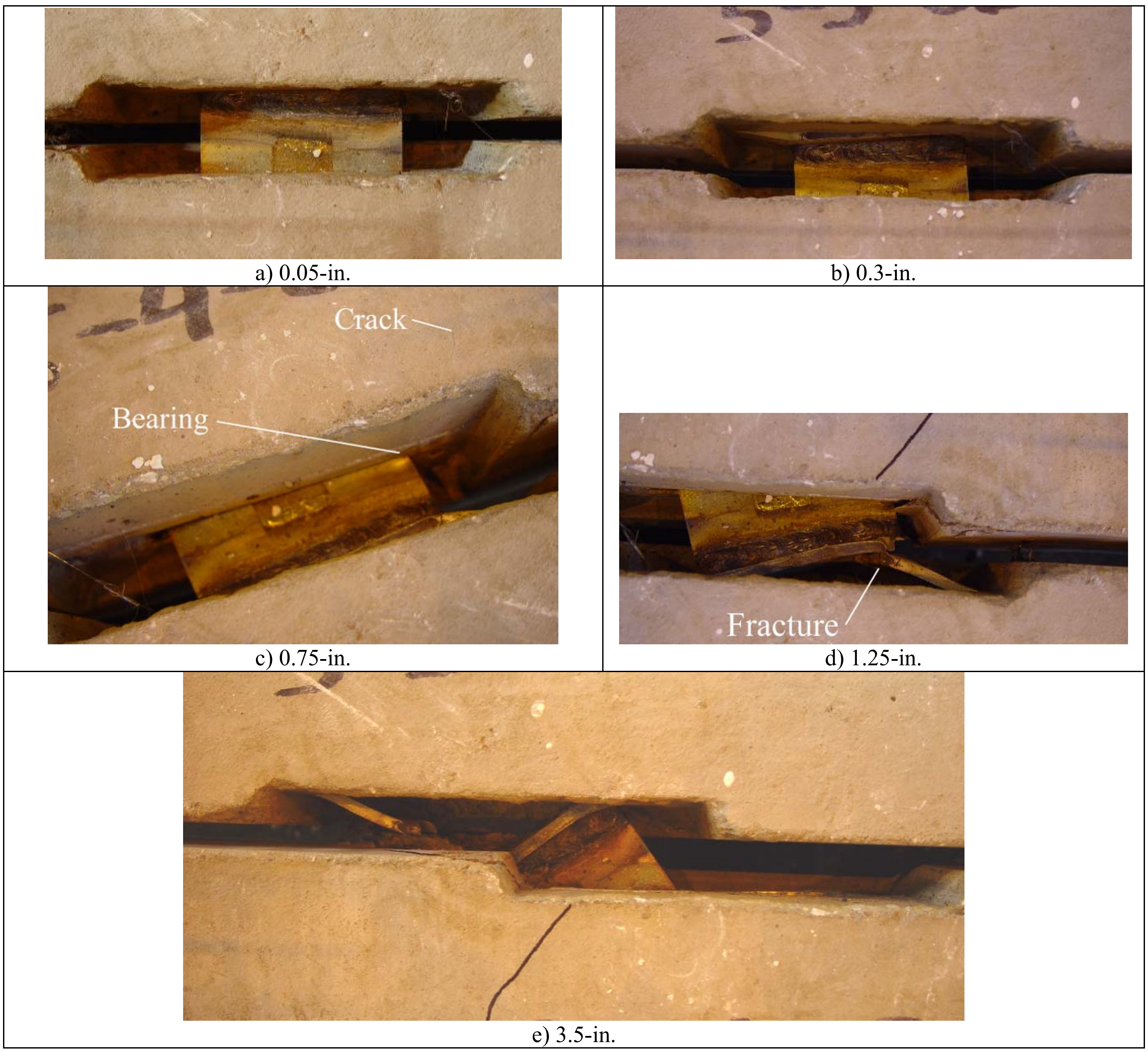


Figure 10: Force and shear displacement (monotonic shear w/ 2 welds)

TEST A3: ERECTOR CONNECTOR 2 MONOTONIC SHEAR DEFORMATION W/ TENSION ΔT = 0

The performance of the Erector Connector subjected to monotonic shear is presented in this section. ***The connector is welded on side B only***. The panel was subjected to a monotonic shear displacement with the tensile displacement restrained, ΔT=0. The connector exhibited an initial shear resistance from the tension leg of connector B. This was followed by bearing of the embedded slug on the faceplate of connector A which produced a splitting crack above the compression connector leg. Fracture of the tension leg of connection B followed resulting in a loss in load carrying capacity. Fracture of the tension leg of connector B progressed slowly resulting in maintenance of shear resistance. After the faceplate was fully fractured shear was maintained through bearing of the slug. The observed key events and the corresponding displacement level are presented in Table 7. The photos of the damage are presented in Figure 11. The global force deformation response and backbone curve are presented in Table 8, and Figure 12.



a) 0.05-in.

b) 0.3-in.

c) 0.75-in.

d) 1.25-in.

e) 3.5-in.

Figure 11: Damage state at increasing shear deformations

Table 7: Key Test Observations Erector Connector - 1 Weld (Monotonic Shear)			
Event #	Shear Δ [in.]	Tension Δ [in.]	Event Description
1	0.30	0.00	Noticeable bending of faceplate on panel B
2	0.75	0.01	Bearing of slug on panel A, yielding of faceplate on panel B
3	1.00	0.01	Concrete cracking around faceplate on panel A
4	1.25	0.02	Initiation of fracture of tension side of faceplate on panel B
5	1.75	0.04	Spalling on underside of panel A
6	2.00	0.05	Bearing of panel A faceplate on panel B
7	3.00	0.11	Complete fracture of faceplate on panel B

Table 8: Experimental Results Backbone Curve Erector Connector - 1 Weld (Monotonic Shear)			
Step	Shear Displacement [in.]	Shear Force [kips]	Axial Force [kips]
	0.143	6.38	-1.25
	0.337	11.92	-3.65
Max Load	0.838	21.43	-9.80
	1.128	8.68	-4.60
	1.395	10.46	-5.76
	1.828	5.97	-2.89
End of Test	4.233	4.61	-2.91

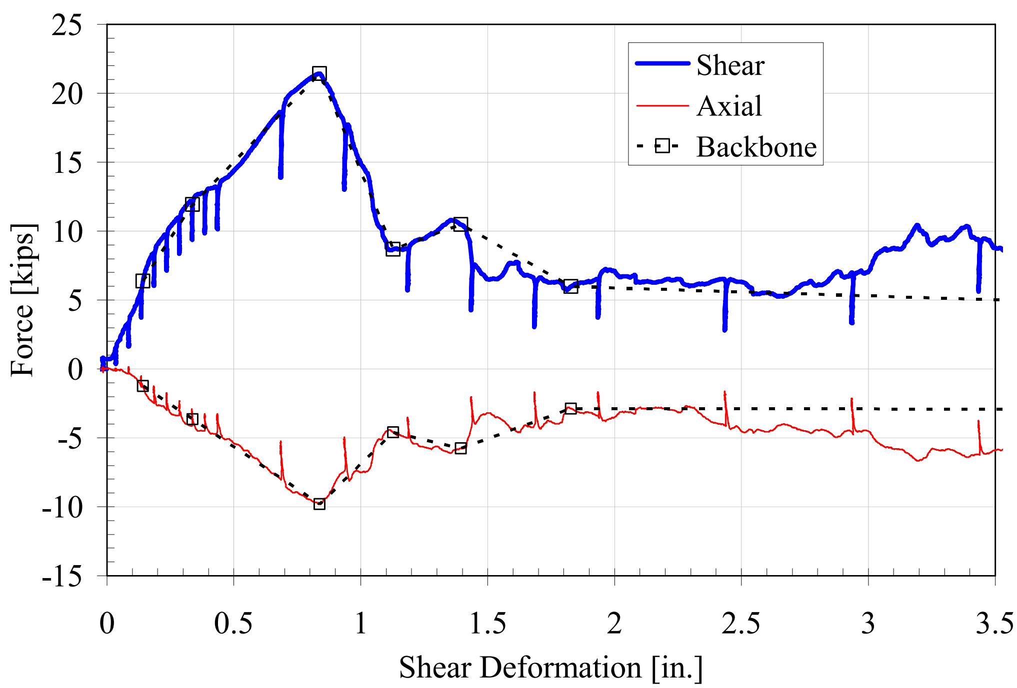
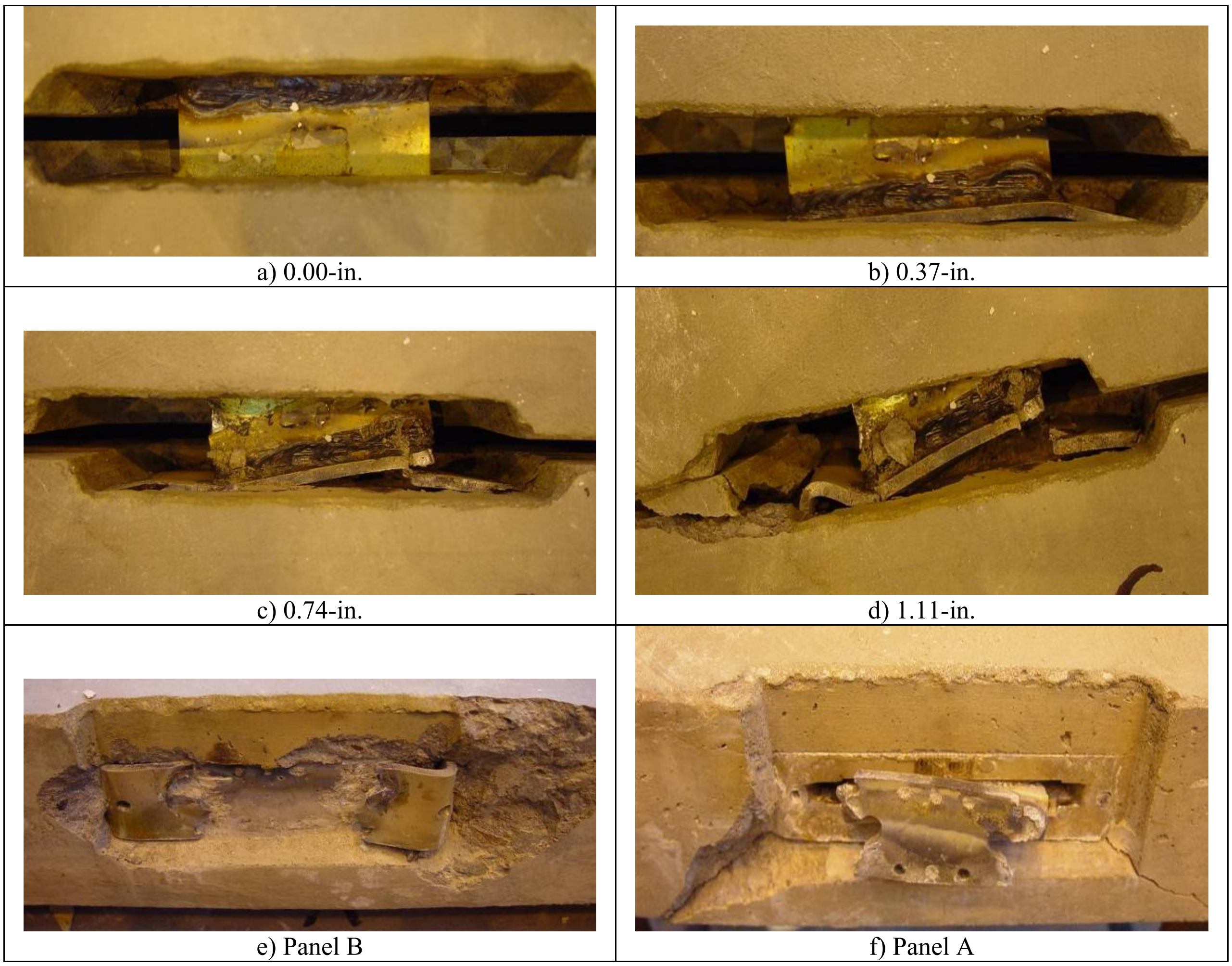


Figure 12: Shear force and displacement (monotonic shear w/ 1 weld)

Test A4: Erector Connector Under Cyclic Shear Deformation w/ Tension ΔT = 0

The performance of the Erector Connector subjected to cyclic shear is presented in this section. ***The connector is welded on side B only***. The panel was subjected to a cyclic shear displacement with the tensile displacement restrained, ΔT=0. Shear deformation resulted in tension demand on one leg of connector B and of the slug bearing on connection A. This produced rotation of the faceplate of connection B. Load reversal caused the faceplate to rotate in the opposite direction and produce tension on the opposite leg. The cyclic loading produced yielding in tension followed by buckling in compression. The repeated cyclic compression and tension resulted in fracture of the faceplate of connector B on either side of the slug. In addition, the bearing action of the slug within connector A produced cracking below the connector. The observed key events and the corresponding displacement level are presented in Table 9. The photos of the damage are presented in Figure 13 through. The global force deformation response and backbone curve are presented in Table 10, Figure 14 and Figure 15.



a) 0.00-in. b) 0.37-in.
c) 0.74-in. d) 1.11-in.
e) Panel B f) Panel A

Figure 13: Damage states cyclic shear with 1 weld

Table 9: Key Test Observations Erector Connector – 1 Weld (Cyclic Shear)			
Event #	Shear Δ [in.]	Tension Δ [in.]	Event Description
1	0.555	0	Spalling on connector B
2	-0.74	0	Fracture on connector B
3	1.11	0	Through fracture on connector B
4	-1.11	0	90% fracture on other side of connector B
5	-1.48	0	Complete fracture of connector B
6	1.48	0	Test Stopped

Table 10: Experimental Results Backbone Curve Erector Connector – 1 Weld (Cyclic Shear)		
Event	Shear Displacement [in.]	Shear Force [kips]
	0.855	17.07
Positive Peak	0.736	19.18
	0.555	16.01
	0.370	12.93
	0.276	10.79
	0.183	8.14
	0.088	2.28
Zero	0.000	-0.18
	-0.097	-1.08
	-0.191	-5.93
	-0.284	-9.10
	-0.377	-10.90
	-0.565	-13.73
	-0.739	-14.96
Negative Peak	-1.076	-16.08

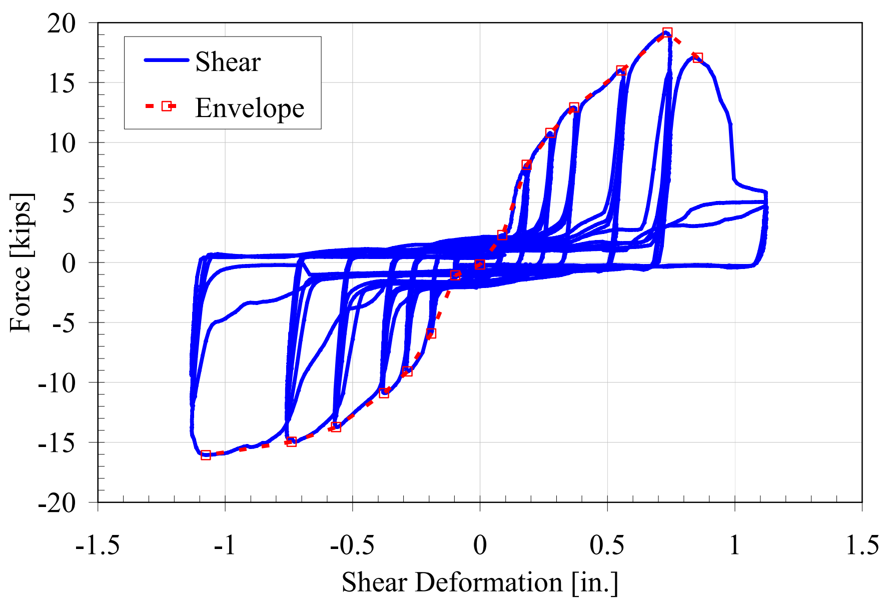


Figure 14: Force and shear displacement cyclic shear with 1 weld

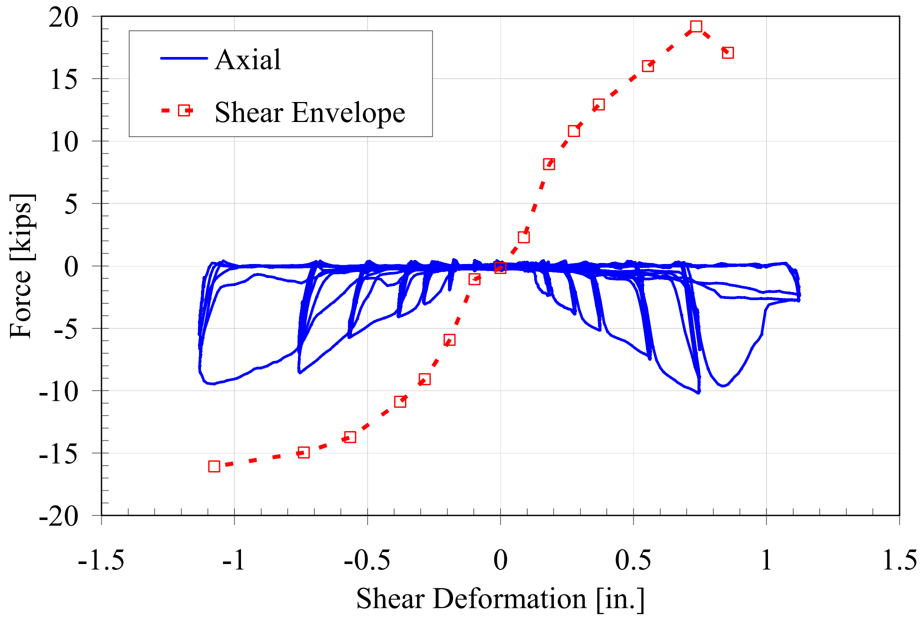
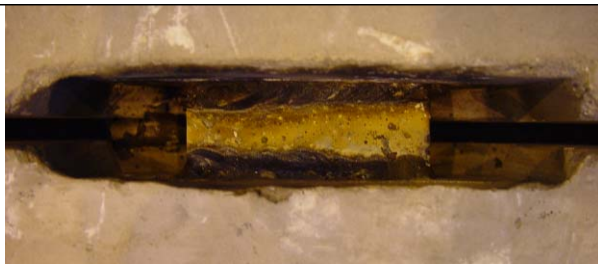


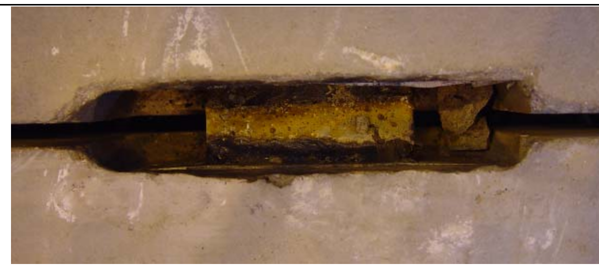
Figure 15: Axial force and shear displacement cyclic shear with 1 weld

TEST A5: ERECTOR CONNECTOR UNDER CYCLIC SHEAR DEFORMATION W/ TENSION $\Delta T = 0$

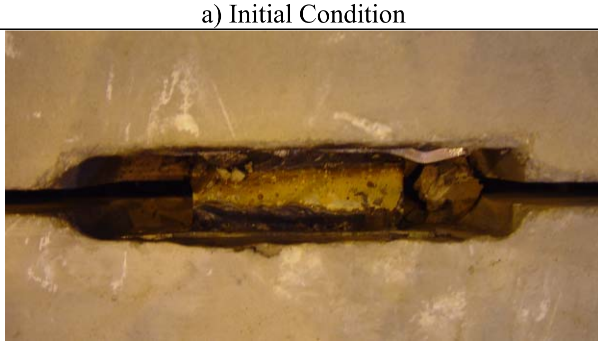
The performance of the Erector Connector subjected to cyclic shear is presented in this section. ***The connector is welded on BOTH sides A and B***. The panel was subjected to a cyclic shear displacement with the tensile displacement restrained, $\Delta T=0$. Shear deformation resulted in tension demand on diagonally opposing legs of the connectors. This produced a rotation of the faceplates of the connection. Load reversal caused the connectors to rotate in the opposite direction and produce tension on the opposite legs. The cyclic loading produced yielding in tension followed by buckling in compression. This produced fracture of the faceplates on both connection A and B. This was followed by spalling above connection A. The observed key events and the corresponding displacement level are presented in Table 9. The photos of the damage are presented in Figure 13. The global force deformation response and backbone curve are presented in Table 10, Figure 14 and Figure 15.



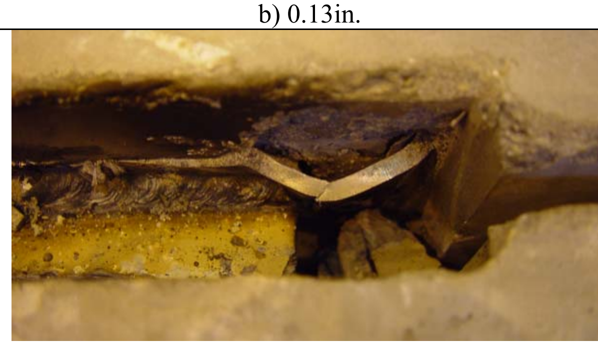
a) Initial Condition



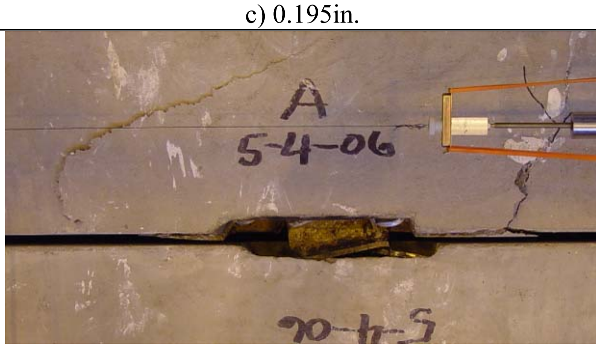
b) 0.13in.



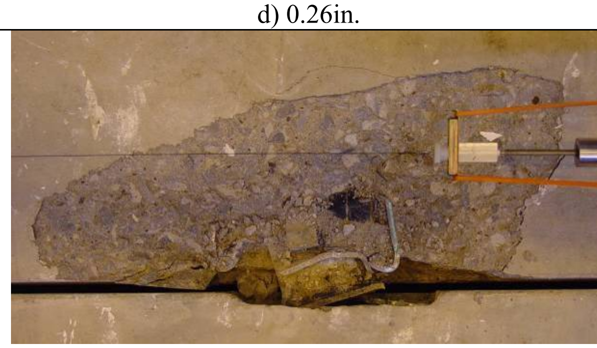
c) 0.195in.



d) 0.26in.



e) 0.78in.



f) 1.04in.

Post test connection A

Post test connection B

Figure 16: Damage states cyclic shear both sides welded

Table 11: Key Test Observations Erector Connector – 2 Welds (Cyclic Shear)			
Event #	Shear Δ [in.]	Tension Δ [in.]	Event Description
1	0.033	0.000	Concrete cracking audible
2	0.098	0.000	Noticeable bending of faceplate on panel A
3	-0.130	0.000	Fracture on faceplate of panel A
4	0.130	0.000	Concrete spalling around faceplate on panel A
5	0.195	0.000	Spalling around faceplate on panel B
7	0.260	0.001	Fracture of faceplate on panel A
8	0.390	0.002	Additional spalling on panel A and faceplate bending
9	-0.520	0.003	Bending of faceplate on panel B
10	-0.520	0.003	Cracking on top of panel A
11	-0.780	0.007	Fracture of faceplate on panel B
12	1.040	0.013	Large spall on panel A
13	-1.300	0.021	Additional spalling
14	-1.560	0.030	End of test

Table 12: Experimental Results Backbone Curve Erector Connector – 1 Weld (Cyclic Shear)			
Event	Shear Deformation [in.]	Shear Force [kip]	Axial Force [kip]
	0.027	7.48	-1.82
	0.058	9.06	-2.71
First Peak	0.092	9.80	-3.04
	0.122	9.09	-2.77
	0.190	5.62	-1.06
	0.257	8.14	-2.73
	0.387	12.66	-5.06
	0.524	15.51	-7.39
Positive Peak	0.710	16.73	-9.07
	1.051	9.87	-4.72
	1.279	3.96	-3.57
	1.575	8.77	-7.28
Negative Peak	-0.503	-17.64	-10.16

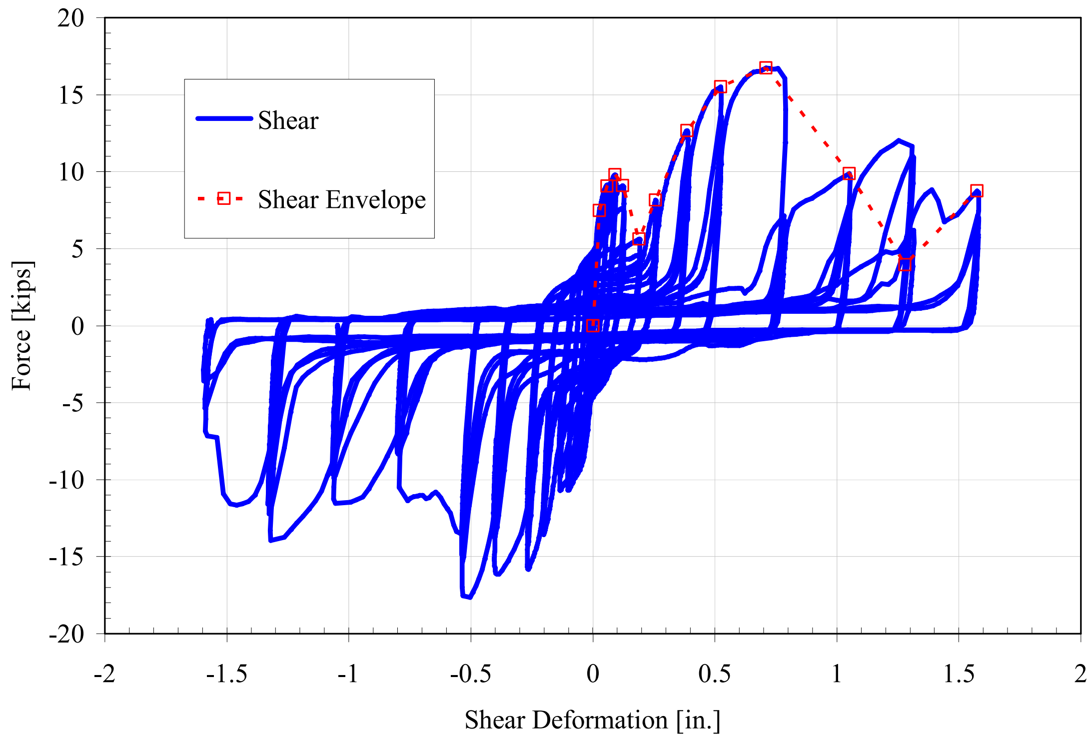


Figure 17: Force and shear displacement CV both sides welded

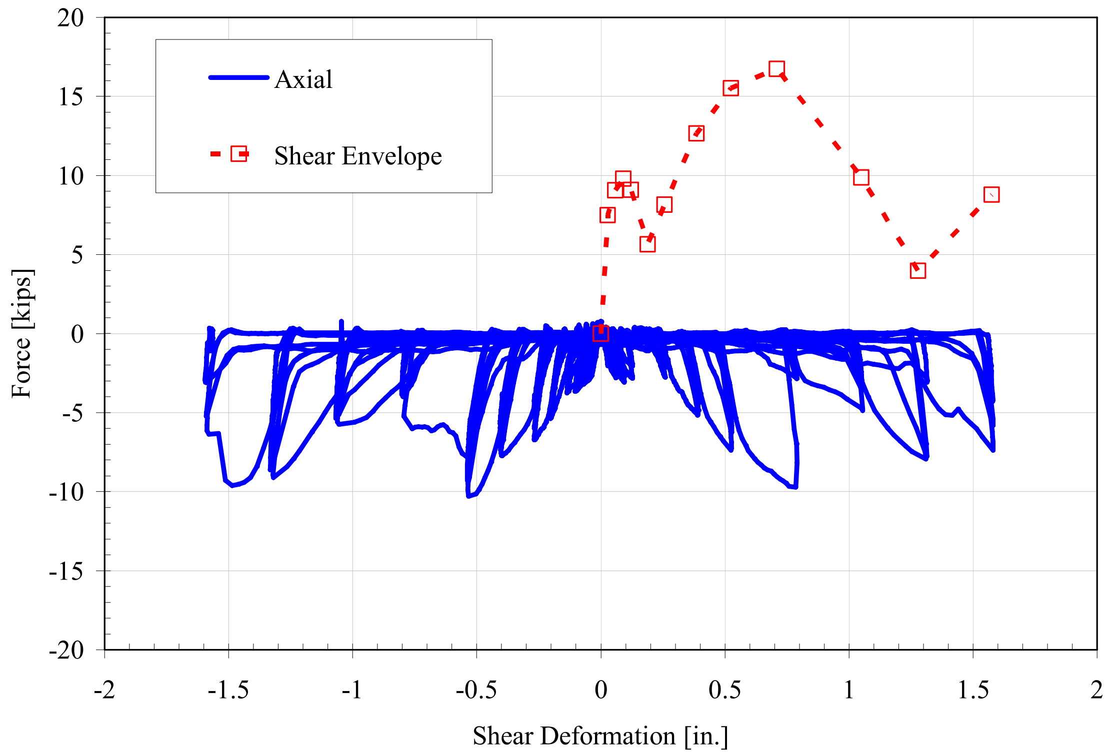
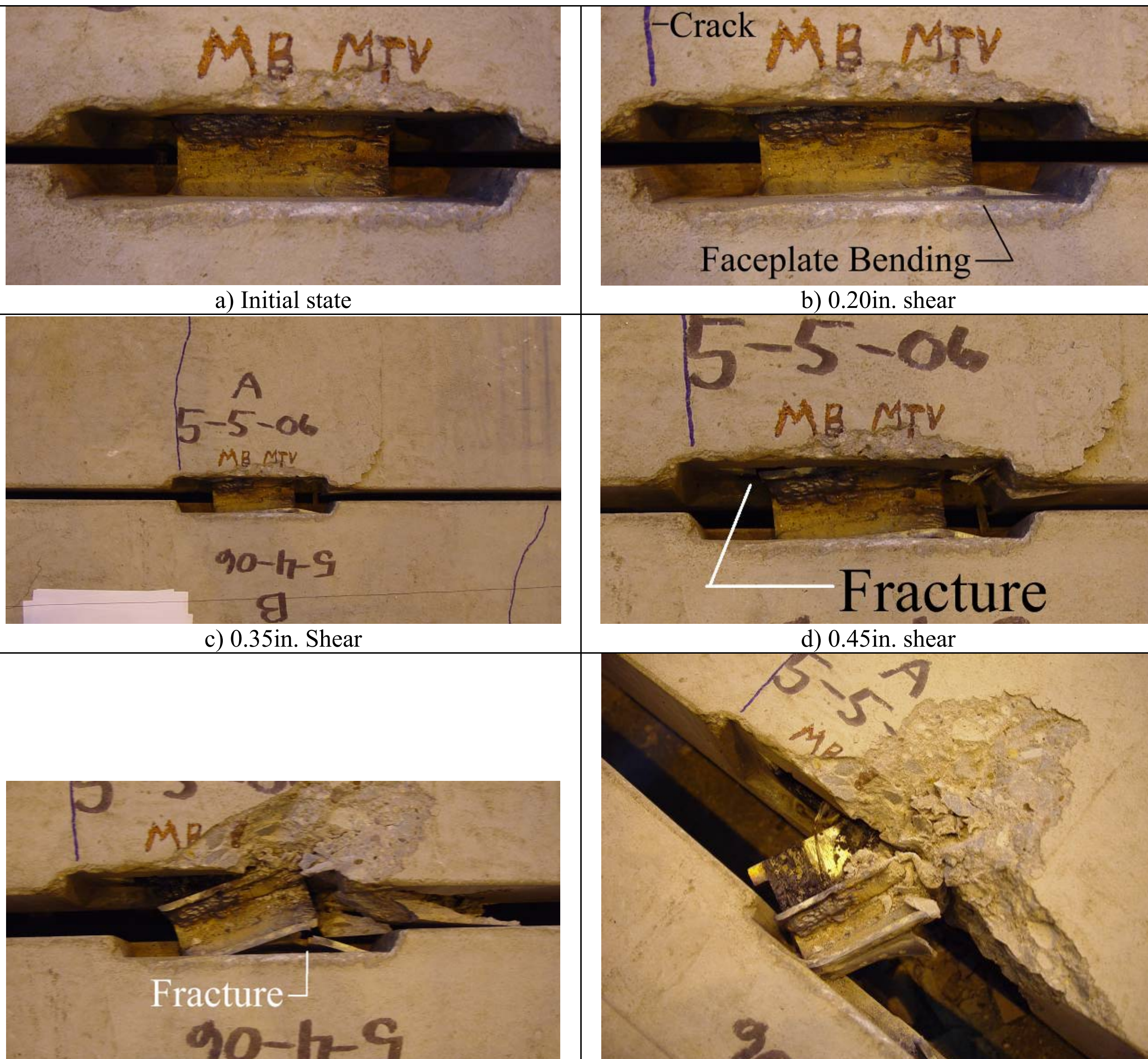


Figure 18: Axial force and shear displacement CV both sides welded

TEST A6: ERECTOR CONNECTOR UNDER SHEAR W/ PROPORTIONAL TENSION DEFORMATION ($\Delta V/\Delta T = 2.0$)

The performance of the Erector Connector subjected to shear with proportional tension is presented in this section. Shear deformation was applied at twice the tension deformation. ***The connector is welded on BOTH sides A and B***. The concrete damage was focused on the panel with Type A connector, however connector fractures occurred on both side A and B. Fracture of connector A occurred first adjacent to the end of the left weld this was followed by fracture of connector B at the adjacent to the right side of the weld. This mechanism formed due to the tension force developed in these two legs as a result of the shear and opening. Once the tension legs were lost the shear was still supported through a compression strut through the opposing legs of the connector. The observed key events and the corresponding displacement level are presented in Table 9. The photos of the damage are presented in Figure 13. The global force deformation response and backbone curve are presented in Table 10, Figure 14 and Figure 15.



a) Initial state | b) 0.20in. shear | c) 0.35in. Shear | d) 0.45in. shear | e) 1.25in. shear | f) End of test

Figure 19: Damage states

Table 13: Key Test Observations Erector Connector – 2 Welds (ΔV/ΔT=2.0)			
Event #	Shear Δ [in.]	Tension Δ [in.]	Event Description
1	0.0000	0.0000	Hairline cracks on panel B
2	0.1000	0.0501	Crack formation on left side of panel A
3	0.1500	0.0753	Noticeable bending of faceplates on both panels
4	0.2500	0.1258	Cracking on panel A above right leg of connector
5	0.3000	0.1511	Additional cracking on panel A above right leg of connector
6	0.4500	0.2275	Fracture of left side of faceplate on panel A
7	1.0000	0.5123	Initial fracture of right side of faceplate on panel B
8	1.7500	0.9128	Spalling on panel A
9	2.0000	1.0494	Complete fracture of right side of faceplate on panel B
10	4.5000	2.4992	End of test

Table 14: Experimental Results Backbone Curve Erector Connector – 2 Welds (ΔV/ΔT=2.0)			
Event	Shear Deformation [in.]	Shear Force [kip]	Axial Force [kip]
-	0.102	9.83	1.59
First peak	0.215	13.91	2.20
-	0.361	10.50	1.87
-	0.831	13.99	8.32
Max Load	1.32	15.20	11.26
-	1.58	13.84	10.47
-	1.86	9.66	8.86
-	3.65	1.98	2.00

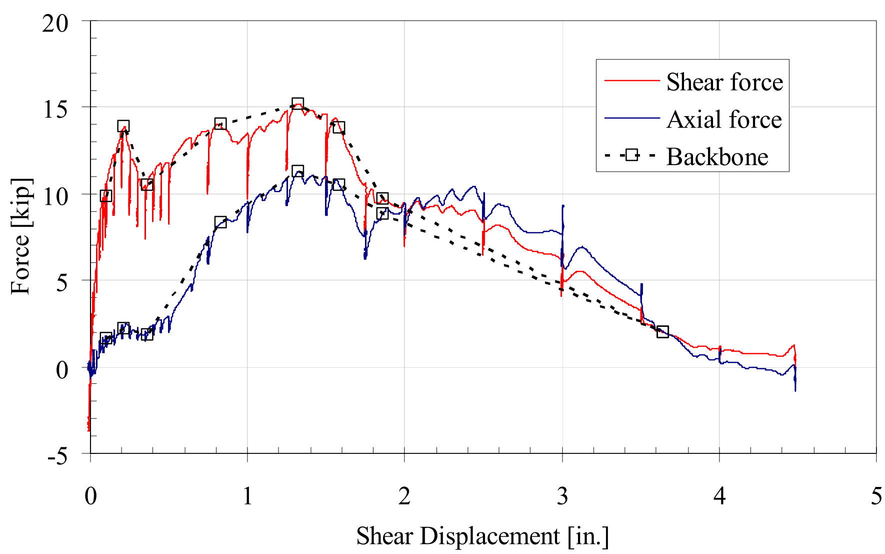


Figure 20: Force and shear displacement MTV

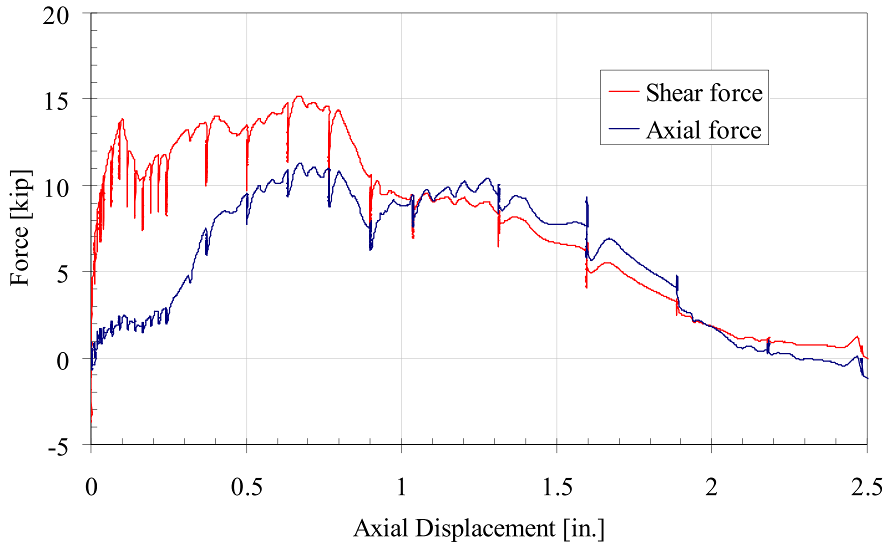


Figure 21: Force and axial displacement MTV